EFFECT OF RATE OF STRAIN ON THE STRENGTH OF REMOLDED SOIL

JULY 1958
NO. 17

by

D. HAMPTON

Joint
Highway
Research
Project

PURDUE UNIVERSITY
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FINAL REPORT

EFFECT OF RATE OF STRAIN ON THE
STRENGTH OF REMOLDED SOIL

TO: K. B. Woods, Director
Joint Highway Research Project
July 9, 1958

FROM: H. L. Michael, Assistant Director
Joint Highway Research Project
File: 6-8-2
Project: C36-14-B

Attached is a final report entitled, "Effect of Rate of Strain on the Strength of Remolded Soil," by Delon Hampton. This study has also been used by Mr. Hampton as partial fulfillment for the degree of Master of Science in Civil Engineering. It was conducted under the supervision of Prof. E. J. Yoder.

Portions of this project were also reported in a paper presented to the 44th Annual Purdue Road School in April 1958.

The report is presented for the record.

Respectfully submitted,

H L Michael

H. L. Michael, Secretary

HLM:acc

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FINAL REPORT

EFFECT OF RATE OF STRAIN ON THE STRENGTH OF REMOLDED SOIL

by

Delon Hampton
Graduate Assistant

Joint Highway Research Project
Project No: C36-14-B
File No: 6-8-2

Purdue University
Lafayette, Indiana

July 9, 1958

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The work reported in this thesis was conducted in the laboratories of the Joint Highway Research Project, Purdue University. Professor E. J. Yoder served as the author's major professor and supervised this project.

The author expresses his sincere appreciation to Professor Yoder for his guidance and encouragement throughout the project. The author expresses his gratitude to Professor K. B. Woods, Head of the School of Civil Engineering, and other members of the staff with whom the author has been associated at Purdue University.

Special thanks are extended to Mr. L. E. Witek who had charge of the preparation of the soil and the molding of all specimens. He also contributed practically all the drafting required in this report -- the remainder was done by Mr. B. R. Bradley.

Photographs were taken by Mr. E. L. Black.

TABLE OF CONTENTS

Page

LIST OF TABLES v

LIST OF FIGURES vi

ABSTRACT vii

INTRODUCTION 1

PURPOSE AND SCOPE 4

DEFINITIONS 5

REVIEW OF LITERATURE 7

DESCRIPTION OF MATERIALS

General 14
Soil Properties 15

MOISTURE - DENSITY RELATIONSHIPS 16

APPARATUS AND PROCEDURE

Preparation of the Soil 19
Molding 19
Unconfined Compression Tests 25
Collection of Data and Instrumentation 27
Presentation of Data 33

DISCUSSION OF PROCEDURE 35

RESULTS

Unconfined Compressive Strength 38
  Silty Clay 41
  Clay 45
Modulus of Deformation 51
  Silty Clay 52
  Clay 53
  Comparison of Clay and Silty Clay 59

Page

CONCLUSIONS 62

DISCUSSION OF POSSIBLE APPLICATIONS

Stability of Slopes Subjected to Earthquakes 64
Applications to Highway and Airport Design 65

RECOMMENDATIONS FOR FURTHER RESEARCH 69

BIBLIOGRAPHY 70

LIST OF TABLES

Page

1. Summary of Atterberg Limits 15
2. Comparison of Kneading and Dynamic Compaction 17
3. Summary of Results of Strength Tests - Silty Clay 43
4. Summary of Results of Strength Tests - Clay 48
5. Summary of Modulus of Deformation Data - Silty Clay 55
6. Summary of Modulus of Deformation Data - Clay 58

LIST OF ILLUSTRATIONS

Page

1. Preparation of the Soil 20
2. Harvard Compaction Apparatus 21
3. Compaction Apparatus in Use 22
4. Equipment Used for Slow and Medium Transient Tests 26
5. Close-up View of Loading Frame and Measuring Instruments . . 28
6. Fast Transient Test Apparatus 29
7. Schematic Diagram of Test Set-up 31
8. Typical Oscillograms 32
9. Moisture Content vs. Dry Density, Unconfined Compressive Strength, and Modulus of Deformation - Silty Clay 39
10. Moisture Content vs. Dry Density, Unconfined Compressive Strength, and Modulus of Deformation - Clay 40
11. Rate of Strain vs. Unconfined Compressive Strength - Silty Clay 42
12. Dry Density vs. Unconfined Compressive Strength - Silty Clay 46
13. Rate of Strain vs. Unconfined Compressive Strength - Clay . . 47
14. Dry Density Vs. Unconfined Compressive Strength - Clay . . . 50
15. Rate of Strain vs. Modulus of Deformation - Silty Clay . . . 54
16. Rate of Strain vs. Modulus of Deformation - Clay 57

ABSTRACT

Hampton, Delon. M.S.C.E., Purdue University, June 1958. Effect of Rate of Strain on the Strength of Remolded Soil. Major Professor: E. J. Yoder.

This thesis reports the results of a laboratory investigation of the effects of rate of strain on the strength of remolded soil. Two soils were selected for purposes of this study: (a) A red-colored clay derived from limestone and pedologically classified as Frederick, and (b) a brown glacial silty clay, pedologically classified as Crosby "B" horizon. These soils were selected primarily on the basis of their difference in plasticity.

Rate of strain was considered the most important variable and it was studied from 0.55 in./min. to 1780 in./min.. The factors of moisture content and dry density were also of prime importance. Consequently, three compactive efforts were used and specimens were molded and tested on both sides of the optimum moisture content of each compactive effort.

The unconfined compressive strength test was used as the strength criteria, and the effect of the aforementioned variables upon the ultimate strength and modulus of deformation of the samples, as determined by this test, are reported. Also, application of the relationships obtained to problems in slope stability, airport pavement design, and highway pavement design are presented in this thesis.

INTRODUCTION

The fact that the compressive strength of a soil is a function of the time required to reach the failure load has long been recognized. However, this area of soil mechanics has not been extensively explored and much work remains to be done, in order that the effects of this phenomenon can be properly evaluated. The specific areas where this information would be of the greatest benefit are as follows:

(a) stability of slopes subjected to earthquakes and other forms of transient loading,

(b) transmission of forces from explosions through soils,

(c) design of highway pavements, and

(d) design of airfield pavements.

Stability of Slopes

In areas where there is a possibility of earthquakes, it is of the utmost importance to investigate the stability of slopes, both natural and man-made, under transient conditions. Such an investigation is especially necessary when failure of the slope in question would be disastrous. Earthquake shocks induced in the earth represent transient loading conditions and critical slopes in an area should be designed and analyzed on this basis.

Transmission of Forces from Explosions

The transmission of forces from explosions through soil, due to

the short time of loading, is another example of where structures subject to such forces should be designed and analyzed on the basis of transient loading tests. Such a rigorous study would be valid only for critical military installations; consequently, this cause of transient loads in soil will not be given further consideration.

Design of Highway Pavements

It is generally recognized that the stress-strain characteristics of pavements are a function of the rate of strain. This can be readily seen by observing the condition of pavements at critical sections along a given route.

From such studies, it has been found that road intersections, uphill grades and other sections where traffic is required to move slowly, or to stop, show distress much more rapidly than their counterparts i.e. open road, free of stops, and down-hill grades. This is believed to be caused by the difference in speed of travel over the aforementioned sections. As a result, there is much need of a comprehensive study on the effect of rate of strain on the behavior of soils.

On the basis of the above, a different design criteria, as well as specifications for the materials used in construction, should be applicable to each case. If there is a great difference in the design based on the two methods of loading a substantial savings in cost would be obtained by altering the design accordingly.

Design of Airfield Pavements

Because of the high speed at which airplanes travel over runways, they are subjected to transient loading conditions which are vastly

different from the relatively static conditions to which the aprons, taxiways, and ends of runways are subjected. Hence, it is necessary to evaluate the difference in stress-strain characteristics under both transient and static loading, for all materials involved, in order to obtain the most economical as well as the best design.

PURPOSE AND SCOPE

The previous discussion points out that the effect of rate of strain on the strength of soil is important. Consequently, the primary purpose of the research reported herein was to investigate the strength properties of a clay and silty clay under conditions of transient loading. Specifically the aim was to attempt to ascertain the relationship between rate of strain and unconfined compressive strength at various moisture contents and densities. Also, it was hoped to relate the aforementioned variables to the modulus of deformation.

The number of variables involved in a study of this type are quite large. Since this research was, relatively, a pioneer effort the principal guide in selecting the primary study variables had to be left to the discretion of the personnel involved.

Rate of strain was considered the most important variable, and it was studied from 0.55 in./min. to 1780 in./min.. Soil texture was a second variable and two fine-grained soils of significantly different characteristics were chosen - a silty clay and a clay. All soils are native to Indiana.

The factors of moisture content and dry density were also of prime importance. Three compactive efforts were used and specimens were molded and tested on both sides of the optimum moisture content (O.M.C.), of each compactive effort.

DEFINITIONS

It is assumed that the reader is familiar with terms pertaining to conventional strength tests on soils. However, before proceeding further he should familiarize himself with the following terms:

(a) A Slow Transient Compression Test is one in which the rate of strain lies within the range of 0.45 in./min. to 0.6 in./min.

(b) A Medium Transient Compression Test is one in which the rate of strain lies between 11.0 in./min. and 15.5 in./min.

(c) A Fast Transient Compression Test is one in which the rate of strain is greater than 250 in./min.

(d) Time of Loading is defined as the difference in time between the start of a loading test and the time at which the maximum compressive stress is reached.

(e) Compressive Stress is the axial load per unit of cross-sectional area of a test specimen. In computing the compressive stress, a correction was applied to the area assuming that no volume change took place, i.e. change in area assumed proportional to the change in height.

(f) Modulus of Deformation is a secant modulus defined as the slope of a line from the origin through the point on the stress vs. strain curve at which the stress is one-half of the compressive strength. Or, if the initial part of the

stress vs. strain curve is straight, it is the slope of this portion of the curve.

(g) Rate of Strain is defined as the deformation at failure divided by the time required to reach failure.

(h) Strength Ratio, S_u, is defined as the ratio of the strength, for a given rate of strain, to that for a slow transient test at the same moisture content and for the same compactive effort.

(i) Modulus of Deformation Ratio, M_D, is defined as the ratio of the modulus of deformation for a given rate of strain to that for a slow transient test at the same moisture content and for the same compactive effort.

(j) Strength, in this report will always apply to the axial load required to produce failure in an unconfined compression test.

It should be noted that the above definitions may be in variance with those by other investigators, but were adopted for this study because they give the best representation of the data.

REVIEW OF LITERATURE

The effect of time of loading on the strength and modulus of elasticity of metal and concrete has been extensively investigated. Also, a considerable amount of work has been done on rock and wood, but little on soil.

The standard methods of measuring the time effect, in previous investigations, was either by the time of loading, the rate of loading, or the rate of strain. In general, the method utilized was determined by the characteristics of the testing machines, i.e. with the equipment available which of the aforementioned methods of measurement can be used most accurately and efficiently. Nevertheless, whenever the time-stress-strain relationships are known, any one of the above ways of measuring the time effect may be converted to either of the other two.

Comparisons between the moduli determined by vibration tests and those determined by static tests may be found in much of the work on the effect of time on the modulus of elasticity. The use of vibrations, called the dynamic or sonic method, is based on the measurement of the velocity of pulse waves through the material tested. The velocity of these waves can be measured directly or it can be computed from the resonant frequency of some fundamental mode of vibration when the specimen is subjected to vibrational impulses (1)*.

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* Numbers in parenthesis refer to references in the Bibliography.

In a dynamic test the modulus is determined for stresses which are very small compared to those generally used in determining the static modulus. Consequently, the comparisons between static and dynamic moduli are significant only if the shape of the stress strain curve is taken into consideration.

With the above as background material a summary of previous investigations is presented below.

Concrete

Abrams (2) in 1917, reported his investigations of the effect of rate of strain on the strength properties of 6 in. x 12 in. concrete cylinders. The rate of strain was varied between 0.006 and 0.15 in. per minute and he concluded that the compressive strength of concrete increases with increasing rate of strain, up to 0.12 in. per minute, but with no further increase apparent at faster rates. Abrams also found that if part of the load is applied at a fast speed and the remainder at a slow speed, the ultimate strength of the concrete is not changed even though as much as 88 per cent of the total load be applied at the fast speed.

Evans (3), in 1942, reported tests which indicated that the compressive strength of concrete cubes remains approximately constant for times of loading greater than about 0.06 seconds. However, the compressive strength of the cubes for times of loading between 0.06 seconds and 0.001 seconds increased 40 to 50 per cent.

Later, Jones and Richart (4) tested the effect of the rate of loading on the compressive strength of 6 inch diameter and 12 inch long cylinders. The loading rates ranged from 0.1 to 2000 lbs. per square

inch per second and it was found that an increase in rate of loading was accompanied by an increase in strength approximately proportional to the logarithm of the loading rate. Since the aforementioned loading rates correspond to a range of time of loading of 15,000 seconds to 1 second the work of Jones and Richart and that of Evans are contradictory.

Jones and Richart also showed that the secant modulus at both 50 per cent and 90 per cent of the maximum load showed a tendency to increase with an increase in rate of loading. Tests by Evans (3), showed an increase in the secant modulus of 7 per cent for a specimen loaded to a stress of 2000 pounds as compared with the same specimen loaded slowly. The latter information was derived from a limited number of tests on concrete columns.

Thompson (5) also investigated the effect of rate of loading on the modulus of rupture of concrete beams. His work showed that beams tested to failure in 0.015 seconds had a modulus of rupture approximately 74 per cent greater than when tested to failure in 10 seconds.

Metals

Manjoine and Nadai (6) conducted tests prior to 1939 on the effect of time of loading on the strength of materials. Their work indicates that the strength of some metals increases with a decrease in the time of loading. It also showed that the curve representing the general speed law of plastic deformation at a given temperature is characterized by an inclined straight line in the semi-logarithmic plot at the intermediate range of speeds (10^{-5} to 10^{2} inches per second).

The metals tested, by the above, were steel, copper, aluminum and a very pure iron. These specimens were tested in tension at varying

temperatures as well as different rates of loading.

As regards the moduli of elasticity of steel, copper, aluminum, lead, nickel and tungsten, work by Ide (7), Sears (8) and King (9) showed that it is independent of time of loading. Also, King (10) studying copper and molybdenum found that the shear modulus of these materials is not influenced by the time of loading.

Wood

Brokaw and Foster (11) made an investigation of the effect of time of loading on the strength and stress-deformation properties of Sitka spruce and Douglas fir. It was observed that an increase in rate of loading increases the ultimate compressive and flexural strength but has little effect on the modulus of elasticity (1).

Rock

Ide (12) compared the dynamic and static moduli of elasticity and found for samples of granite, olivine, diabase, peridotite, serpentine and quartzitic sandstone that the dynamic modulus is from 5 to 12 per cent greater. However, for sandstone, tuff, and gneiss, the dynamic modulus is from 20 to 40 per cent greater than the static modulus.

Evans (13) discovered that the secant modulus of sandstone and granite increases with decreasing time of loading (1). For example, by loading granite to a stress of 4000 lbs. per sq. in. in 0.09 seconds the modulus can be increased 8 per cent. For sandstone, loading to a stress of 3,200 lbs. per sq. in. in 0.1 seconds, the secant modulus is approximately 20 per cent greater than that determined for a slow test.

Casagrande and Shannon (1) performed triaxial compression tests

on Cucaracha Rock, from the Canal Zone, and the results show that the modulus of deformation increases with decreasing time of loading. At this point it should be emphasized that moduli of rock depends on the amount of fracturing and the condition of the slickenslides as well as the chemical composition of the individual specimens.

The previously mentioned authors have published the only information available on the effect of varying rates of loading on the strength of rock. They showed that the compressive strength for fast transient tests (time of loading 0.02 sec.) is about 1.6 times the strength in static tests (time of loading 10 minutes).

Soil

In connection with the design of the Panama Canal several triaxial compression tests were made to determine the effect of time of loading on the compressive strength of Gatun black muck (14). In one test, with a time of loading of approximately 90 seconds, the compressive strength was about 40 per cent greater than in 4 other tests in which failure was obtained in about 1 hour.

A series of triaxial compression tests, on remolded-consolidated Boston blue clay, was reported by Taylor. The rate of strain ranged between 1 and 0.0005 per cent per minute and the compressive strength for the fastest test was approximately 20 per cent greater than that for the slowest test (1).

Casagrande and Shannon (1) found that the modulus of deformation of clays for fast transient tests was approximately twice as great (this was for both unconfined and consolidated quick tests) as that for slow tests. Also, the modulus of deformation of Manchester sand increased

slightly with decreasing time of loading i.e. the average value for static tests is about 300 kg. per sq. cm. and for fast transient tests about 400 kg. per sq. cm.

Furthermore, for all tests on clays the transient compressive strength was greater than the static compressive strength. The fast transient compressive strength, taken at a time of loading of 0.02 seconds, ranges between 1.4 and 2.6 times greater than the 10 minute static compressive strength. The above holds true regardless of whether the clay is undisturbed or remolded.

Tests on the Manchester sand indicate that the transient compressive strength of sand in a fast transient compression test was only about 15 per cent greater than the static compressive strength.

Investigations by Seed and Lundgren (15), on the effect of transient loading on the strength of saturated sands supports the above. It was found that the strengths of dense specimens in rapid transient tests was 15 to 20 per cent greater than the strengths of similar specimens in static or slow transient tests.

Whitman (23), also, found that the strength of both undisturbed and disturbed soil specimens was significantly increased by transient loading. This work was done over a wide range of cohesive soils and three sands -- the latter ranging from uniform Ottawa sand to well-graded sands.

On the basis of the data collected Whitman concluded that the effect of rate of strain on cohesive soils should be evaluated from triaxial tests with confining pressures adequate to prevent splitting or shear plane development prior to the occurrence of the peak load. Considering dry sand and moist sand, it was found that increasing the

rate of strain did not produce a significant change in the strength of the material. However, the converse was true for saturated sand and Whitman attributed this to differences in pore water pressures in the slow and rapid tests.

DESCRIPTION OF MATERIALS

Two soils were selected for purposes of this study: (a) a red-colored clay derived from limestone, and (b) a brown glacial silty clay, pedologically classified as Crosby, "B" horizon. These soils were selected primarily on the basis of their difference in plasticity. It was believed that the plasticity characteristics of the soil would be a determining factor in the effect of the rate of strain on these materials.

The clay was from a location in southern Indiana and is the product of the weathering of rock of Mississippian age (16). The soil is rather well graded, highly plastic, very tough near the plastic limit, and has a very high dry strength. In order to obtain the most highly plastic material in the profile, the soil was taken from the layer immediately adjacent to the limestone rock proper (16). The soil would probably be classified pedologically as Frederick, and exists in a profile as shown in Reference (17).

The silty clay is an abundantly distributed material in north-central Indiana. The soil is well graded, of medium plasticity, tough near the plastic limit, and has a high dry strength. In this instance the soil was obtained just outside the corporate limits of West Lafayette. The profile of this Crosby "B" soil is shown in Reference (17).

Soil Properties*

Plasticity: To define the relative plasticity characteristics of the test soils in a quantitative sense, Atterberg limit determinations were made utilizing the apparatus and procedure described by Lambe (18).

TABLE 1

SUMMARY OF ATTERBERG LIMITS

Soil	LL	PL	PI
Silty-Clay	35.4	19.5	15.9
Clay	72.0	28.1	48.9

Specific Gravity: The test on soil solids passing the No. 40 Standard sieve were found to have the following specific gravity values (16).

Soil	Specific Gravity
Silty-Clay	2.69
Clay	2.81

Classification: The above information provided sufficient identification of the soils to place them in classification groups of the Unified Soil Classification System (19).

Soil	Classification
red, limestone residual clay	CH
Crosby "B", glacial silty-clay	CL

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* Data for the silty clay was obtained from Lovell (16).

MOISTURE-DENSITY RELATIONSHIPS

The only relationships studied were those for kneading type compaction with three compactive efforts being used. There is no recognized standard procedure for the equipment used, for the Harvard Miniature Compaction apparatus; however, the compaction procedure recommended in reference (22) was followed. This procedure is presented on page 24 of this report.

It was hoped to obtain densities which approximated the Standard AASHO and Modified AASHO compaction tests as well as a density intermediate between the two. The compactive effort used and the procedure are presented on page 24.

The moisture vs. density curves are shown in Figures 9 and 10. Also, as a comparison between the dynamic and the kneading compaction characteristics of these soils Table 2 is presented.

The data presented, in Table 2, on kneading compaction represents the average values obtained from the three moisture density curves in Figures 9 and 10, for the compactive efforts under consideration. It can be readily seen that the optimum moisture content, and the maximum dry density obtained from the two methods are slightly different, for a given compactive effort. For both soils, the kneading compaction gave values of optimum dry density which were on the average of 2.5 pcf. higher than the Standard AASHO. Also, the kneading compactor gave an O.M.C. which was approximately 3 per cent lower for the silty clay,

TABLE 2

COMPARISON OF KNEADING AND DYNAMIC COMPACTION*

Soil	Compactive Effort	Max (pcf)	O.M.C. (%)
Silty-Clay	Standard AASHO	107.5	19.2
	Kneading Compactor (3 layers at 25 blows)	109.5	16.3
	Modified AASHO	119.3	13.7
	Kneading Compactor (10 layers at 50 blows)	117.0	13.8
Clay	Standard AASHO	91.7	27.8
	Kneading Compactor (3 layers at 25 blows)	94.8	26.0
	Modified AASHO	106.7	19.5
	Kneading Compactor (10 layers at 50 blows)	102.5	22.7

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* Data for dynamic compaction obtained from Reference (16).

and the clay O.M.C. was 1.3 per cent lower than the Standard Proctor O.M.C..

On the other hand, the Modified AASHO test produced dry densities, for both soils, which were greater than those obtained from the kneading compactor, and moisture contents which were less. In the case of the silty clay the difference was approximately 2 pcf. in density but only 0.1 per cent in moisture content. For the clay, the Modified AASHO test produced densities which were 4 pcf. greater and moisture contents which were 3 per cent less. However, though there are measurable differences in moisture content and dry density between the Standard AASHO and Modified AASHO, and the equivalent kneading compaction, they are close enough for the purposes of this study.

The moisture-density curves for the clay are more "peaked" than normal. This condition is probably accentuated by the large moisture scale, but could also be due to the compaction characteristics of the Harvard apparatus.

APPARATUS AND PROCEDURE

Preparation of the Soil

All soil used in this study was air dried, pulverized, and passed through a number 40 U.S. Standard sieve. Predetermined quantities of the soil (hygroscopic moisture content being known) and distilled water were then intimately mixed in a clean pan. Uniformity of mixing was greatly aided by use of an electric vibrator sprayer (see Figure 1) which emitted the water in finely dispersed form. Correct amounts of soil and water were determined on a torsion balance of 0.1 gram sensitivity.

The mixing pan and its contents were then covered with an elastic secured plastic moisture barrier and allowed to "cure" in the constant humidity room for a maximum of 24 hours. The soil was prepared in quantities or batches adequate to provide three samples plus an overrun to cover moisture content checks, normal wastage, and tests which required re-running.

Molding

Subsequent to this period of curing the specimens are molded utilizing the Harvard Compaction Apparatus. This equipment is shown in Figure 2, and from left to right are the tamper, the compaction mold with collar, the extruder, and farthest to the right, the end product of the use of these tools, a compacted specimen.

Figure 3 shows the Harvard Compaction Apparatus in use. In

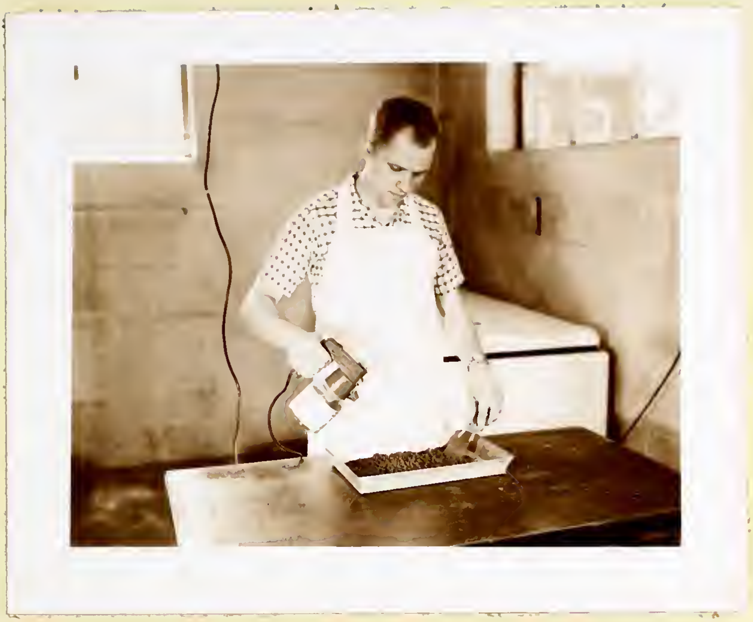


FIG. 1   PREPARATION OF THE SOIL

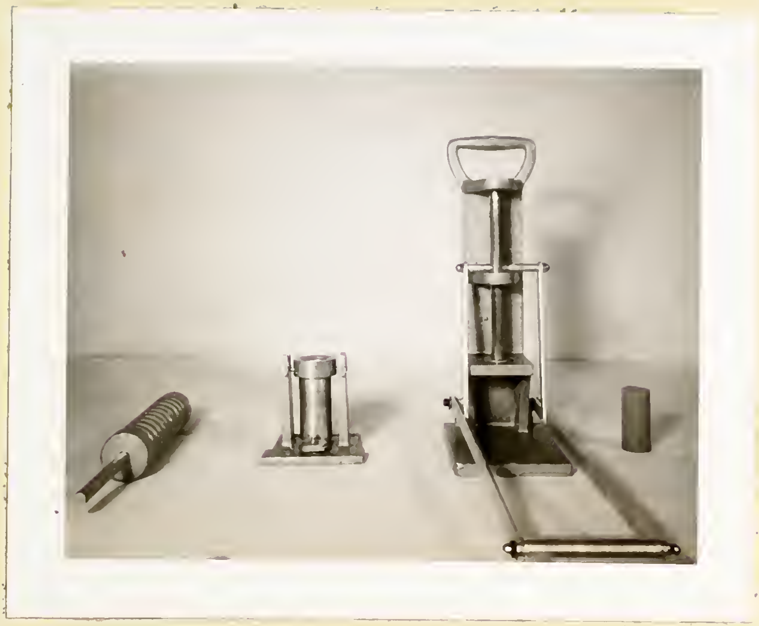
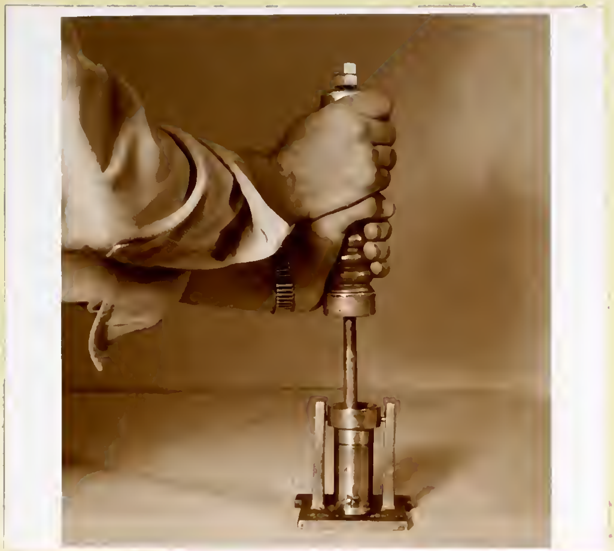
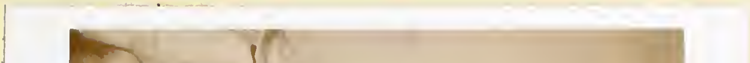


FIG. 2 HARVARD COMPACTION APPARATUS



(a) THE MOLDING PROCESS



(b) EXTRUDING THE MOLDED SAMPLE

FIG. 3 COMPACTION APPARATUS IN USE

Figure 3a, a sample is being molded and in Figure 3b, the molded sample, after removal of the mold collar and leveling the top and bottom of the specimen, is being extruded.

One standard piece of equipment is not shown in the aforementioned figures--a device which holds the mold and soil in place while the extension collar is being removed. It was broken in the very early stages of the tests and was not available for use. However, this piece of equipment was not essential and the molding operation preceeded uninterrupted.

The Harvard Compaction Apparatus produces specimens 2.816 inches long and 1 5/16 inches in diameter. The volume of the compaction cylinder is 1/454th of a cubic foot which means that the weight of a compacted specimen in grams is numerically equal to its unit weight in pounds per cubic foot.

The compactive efforts used were 10 layers at 50 blows per layer, which approximates the Modified AASHO, and three layers at 25 blows per layer which gives dry densities comparable to the Standard AASHO test. The third compactive effort varied depending on the soil being tested. The latter was necessary in order to obtain significant differences in the dry densities obtained from the three compactive efforts. For the Crosby "B" soil the intermediate compactive effort was ten layers at 25 blows per layer while that for the Frederick Limestone soil was ten layers at ten blows per layer.

Each blow was applied through a tamper consisting of a 1/2 inch diameter brass rod equipped with a handle, inside of which there was a 40 pound compressed spring. The operator applied just enough force to

"break" the spring and then released it. In this manner a force of 40 pounds is applied per blow. The action of this compactor is intended to more nearly approximate the kneading action of a sheepsfoot roller than do dynamic compaction methods. Details of the molding technique may be described as follows:

(a) With the mold and collar clamped to the base, place the assembly on a firm compacting surface.

(b) Into the mold place the desired amount of loose soil. For ten layers one slightly heaping teaspoonful of loose soil was required for each layer, and for three layers three heaping teaspoonsful were required per layer. Level the surface by pressing lightly with a plunger.

(c) Insert the tamper in the mold until it is in contact with the surface of the soil, and press down firmly until one feels that the spring is starting to compress (Figure 3a). Release the force and shift the tamper to a new position. Each of the first four tamps was applied in a separate quadrant and adjacent to the mold, and the fifth tamp was in the center, making one complete coverage. This cycle was then repeated until the desired number of tamps was applied.

(d) Add the next layer and repeat the procedure until the required number of compacted layers have been placed. The top layer extended approximately 1/2 inch into the extension collar.

(e) The mold, with collar attached, was then removed from the base. The bond between the soil and the collar was then

partially broken and the collar removed by a rotational movement of the collar with respect to the mold and soil contained therein.

(f) The excess soil was then carefully trimmed away from the top of the mold and from the bottom, if any.

(g) The sample was next removed from the mold with the sample extruder (Figure 3b) and weighed to the nearest 0.01 gram.

(h) Following the weighing operation each specimen was carefully wrapped in aluminum foil, marked for identification, placed in a pan covered with an elastic secured plastic moisture barrier, and placed in the humid room for approximately 24 hours prior to testing.

Unconfined Compression Tests

The unconfined compressive strength tests were run on either the hydraulic loading apparatus or the impact loading apparatus depending upon the rate of strain desired. Figure 4 shows the setup for the slow and medium transient tests utilizing the hydraulic apparatus. The loading frame, containing the specimen and measuring instruments, is shown in the background while the power source and recording device is shown in the foreground.

The power source was an audio oscillator which is shown on the right side of the table in the foreground of Figure 4. Atop the oscillator is a voltmeter which was used to keep the voltage across the terminals of the oscillator at a constant value. The oscilloscope, located to the left of the oscillator, was used as the recording device.

The hydraulic loading apparatus consisted of a constant volume

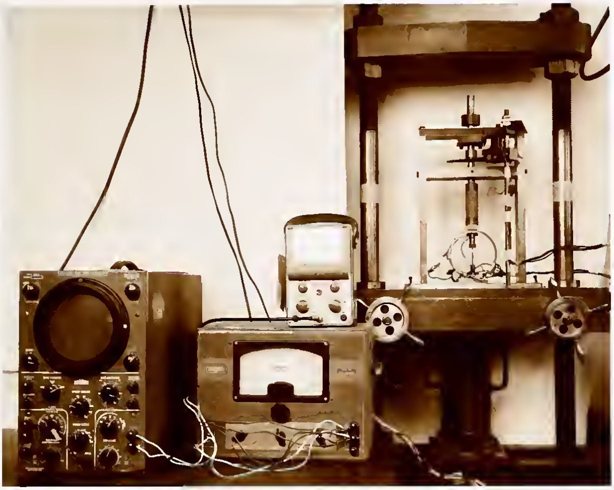


FIG. 4 EQUIPMENT USED FOR SLOW AND MEDIUM TRANSIENT TESTS

hydraulic pump connected to a hydraulic cylinder through values by which the volume of liquid delivered to the cylinder can be controlled. Thus by proper positioning of the valves the speed of the machine could be controlled, and the desired rate of strain obtained. The maximum load which this apparatus can deliver is 20,000 pounds.

Figure 5 is a close-up view of the loading frame and measuring devices used for all tests. It should be noted that for the actual tests the specimens were enclosed in a rubber membrane to retard moisture loss.

The dynamic loading apparatus consists of a ten pound weight dropped 39 inches upon a piston which applied the load to the specimen (see Figure 6). In order to prevent damage to the loading frame and measuring instruments, and to slow down the rate of loading, a spring was inserted atop the loading piston. Another advantage was that the elasticity of the spring caused the weight to bounce, whereupon it was caught before it could again come in contact with the piston.

A hollow pipe, suspended from the top of the wall, was used as a guide for the falling weight--the weight had a hole in the center.

In all unconfined compression tests a rubber membrane was placed around the specimen and secured tightly to the loading head and base. Also, the loading frame was leveled in order that the load would be uniformly applied to the sample.

Collection of Data and Instrumentation

Low voltage differential transformers were employed to measure the load as well as the deformation during the compression process. In this type gage the output voltage of the secondary coils which are ex-

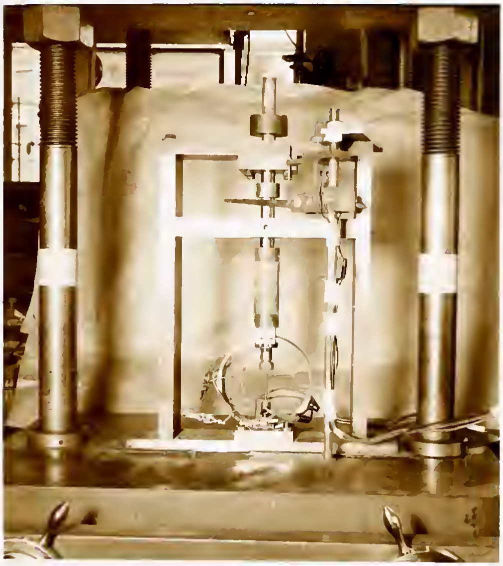


FIG. 5 CLOSE UP VIEW OF LOADING FRAME
AND MEASURING INSTRUMENTS

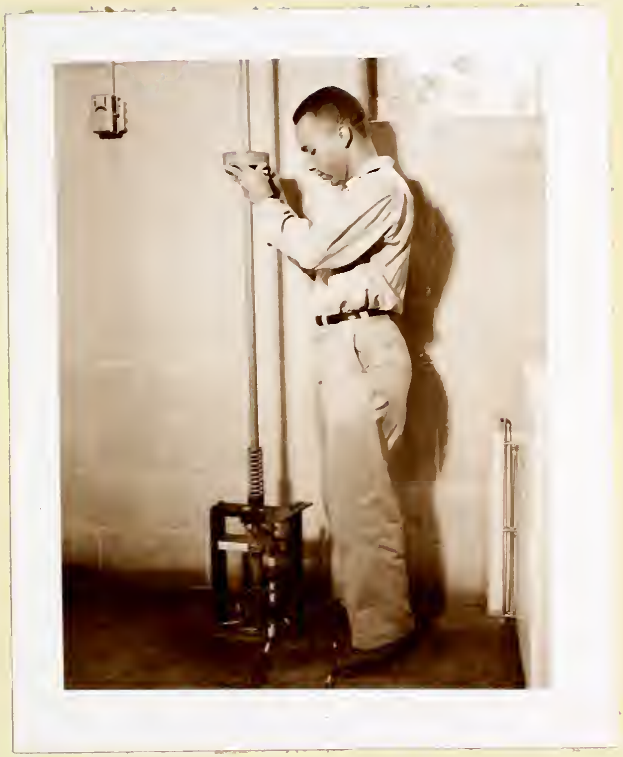


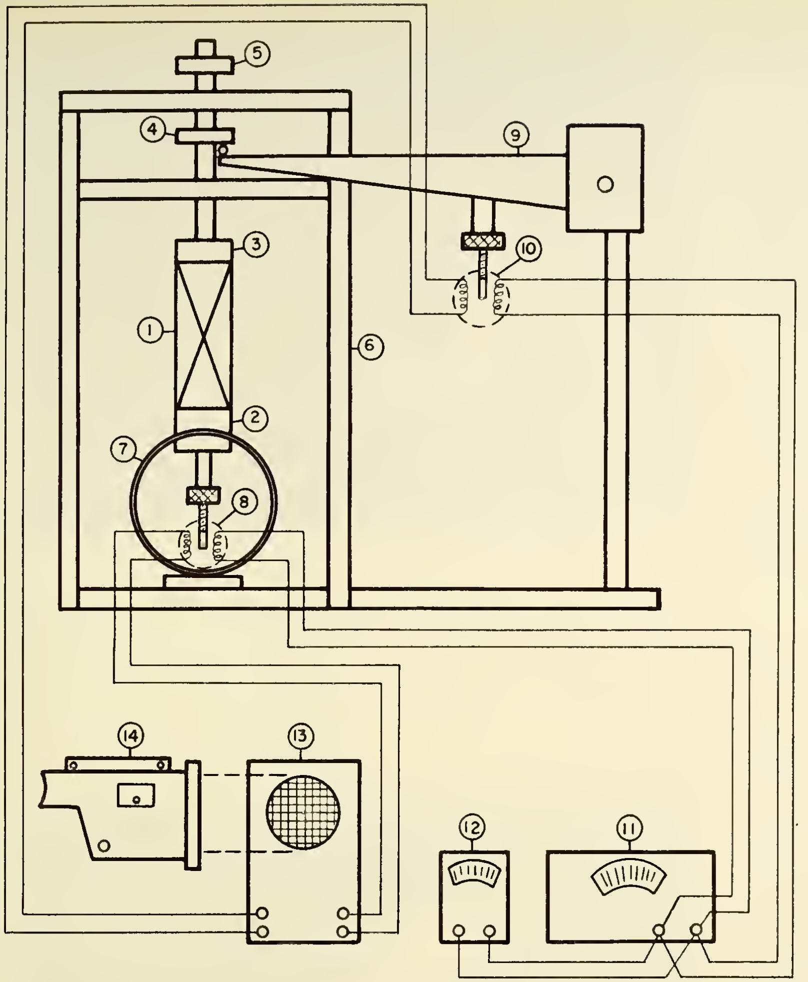
FIG. 6 FAST TRANSIENT TEST APPARATUS

cited by a primary coil is proportional to the displacement of a magnetic core within these coils.

A schematic diagram of the load and deformation apparatus is shown in Figure 7. It can be seen that as the proving ring deforms the position of the core moves relative to the coils. Also, as the arm of the strain gage deflects the core contained therein moves relative to its primary and secondary coils. These movements produce changes in the output voltage, of these two instruments, which can be measured. The operating characteristic of these transformers is ± 0.04 inches and the input voltage was 3.6 volts. A Model 655 Audio Oscillator operating on a frequency of 2000 cps. was used to energize the transformers.

To record the changes in stress and strain during the progress of the test a Dumont Cathode-Ray Oscilloscope was used. This instrument may be described as an electron gun in which a stream of electrons is fired at a screen (1). The stream may be deflected either horizontally or vertically by changing the potential of the plates on either side of the stream. In this study, the deflection of the stream is made proportional to the output of the differential transformers.

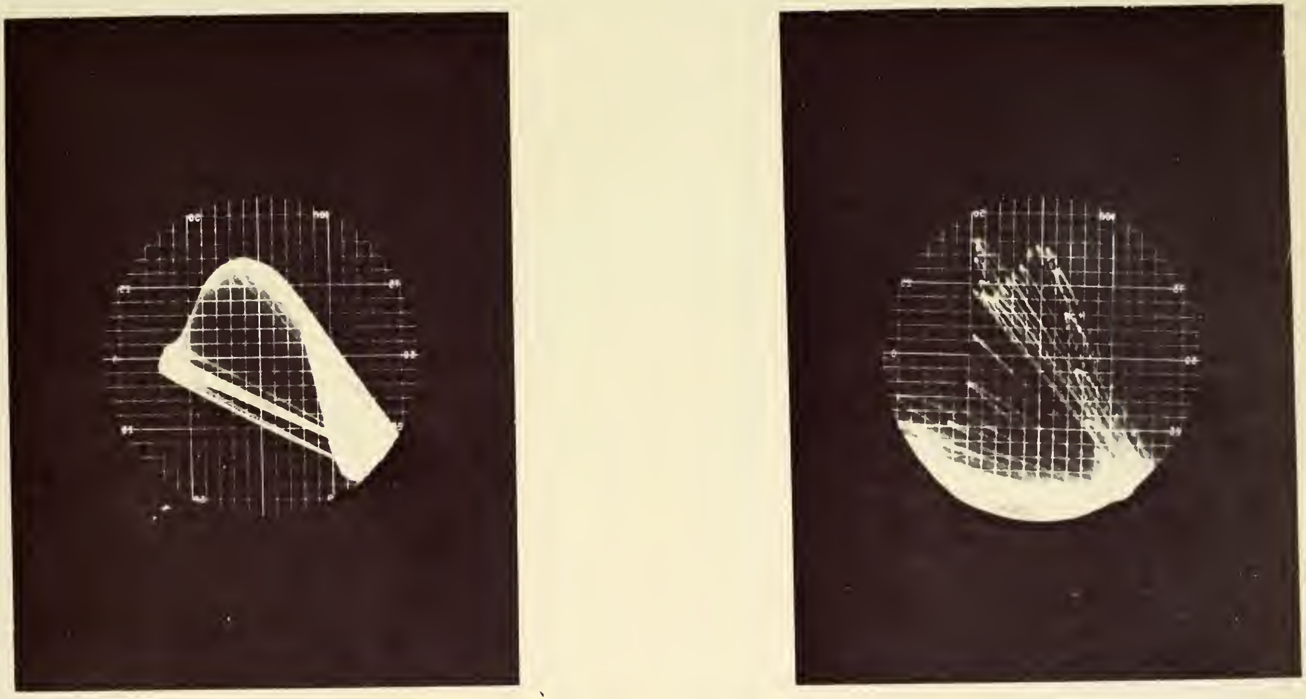
For the medium and fast transient tests the DuMont Oscilloscope Record Camera, Type 297 was used to record the trace on the oscillograph. Operating on the Polaroid-land Principle, it produced a finished oscillogram 60 seconds after exposure (see Figure 8 for typical oscillograms). However, for the slow transient test the load vs. deformation diagram, oscilloscope trace, was drawn directly on the oscilloscope screen, with a grease pencil, during the progress of the test. Utilizing the grid on the face of the oscilloscope this trace was transferred to a data sheet.



① SOIL SPECIMEN
② LOADING HEAD FOR PROVING RING
③ LOADING HEAD FOR SPECIMEN
④ STRAIN DATUM
⑤ STRAIN - STOP
⑥ FRAME
⑦ LOAD GAGE

⑧ DIFFERENTIAL TRANSFORMER
⑨ STRAIN GAGE
⑩ DIFFERENTIAL TRANSFORMER
⑪ AUDIO OSCILLATOR
⑫ VOLTMETER
⑬ OSCILLOSCOPE
⑭ OSCILLOSCOPE RECORD CAMERA

FIG. 7 SCHEMATIC DIAGRAM OF TEST SET-UP



(a.) MEDIUM TRANSIENT TEST

(b.) FAST TRANSIENT TEST

FIG. 8 TYPICAL OSCILLOGRAMS

Calibration

In order to obtain the magnitude of the load and deformation at any instant it was necessary to calibrate the instruments. Although both the load and deformation gages proved stable when operated over considerable periods of time the calibration process was conducted for each series of tests. However, when a series of tests consisted of more than six specimens a recalibration was made following the sixth test.

The load calibration consisted of placing the proving ring, load gage, into a consolidometer and placing known loads upon it. The Y-axis on the oscilloscope grid was used to measure load and when the loads were placed on the ring the movement of the trace was noted. Consequently, knowing the load and corresponding movement of the trace a calibration curve could be constructed.

The deformation calibration consisted of deflecting the lever arm of the deformation gage and noting the movement of the trace along the X-axis. A micrometer mounted on a ring stand produced the known deformations. With this data a calibration curve may be plotted.

Therefore, by obtaining the oscilloscope trace for any test and comparing it to the calibration curves the load and deformation throughout the test could be obtained and a stress strain curve plotted. Also, a record is made of the rate of strain for each test and the moisture content of the specimen determined--in the latter case, the entire specimen is used as a moisture sample.

Presentation of Data

Subsequent to collection of the data the following curves were

drawn: (1) moisture content vs. unconfined compressive strength, for a given compactive effort and rate of strain, (2) modulus of deformation vs. moisture content, for a given compactive effort and rate of strain, (3) unconfined compressive strength vs. rate of strain, for a given moisture content and compactive effort, (4) modulus of deformation vs. rate of strain, for a given moisture content and compactive effort, and (5) dry density vs. strength for a given moisture content and compactive effort. In the aforementioned, average rates of strain representing slow transient (0.45-0.6 in./min.), medium transient (11-15.5 in./min.) and fast transient tests (greater than 250 in./min.), were used.

DISCUSSION OF PROCEDURE

The soil used in this study was part of that processed for prior investigations. Since controlled-humidity storage space was inadequate to permit retaining the sampled soil at or near its field moisture content it was necessary to allow the soil quantities to become air dried. It is recognized that a soil containing clay is irreversibly altered by drying; particularly if the drying is allowed to reach the shrinkage limit (the shrinkage limit is that moisture content below which further drying does not produce additional shrinkage).

Accordingly, the laboratory behavior of the soils is peculiar to their processed condition, and it may differ significantly from the in-situ behavior. If air drying means equilibrium with a relative humidity of about 50 per cent, Lovell (16) has shown that most of the free water remaining in the sample exists at a suction of pF=6 - this is equivalent to a water head of approximately 32,000 feet. If the soil water suction is expressed in terms of the length in cms. of an equivalent water column, the logarithm of this length is the pF scale.

Specimens were molded and tested at moisture contents both greater than and less than optimum for each compactive effort; however, the predominance of the tests were on the dry side for the two lower compactive efforts. The latter was occasioned by the fact that the strengths, on the wet side, were low and there was no advantage in conducting many tests on the wet side. Also, for the soils tested, once optimum

moisture content was passed it became increasingly difficult to satisfactorily compact specimens. The pressure at the end of the tamper was high (80 psi minimum) and at moisture contents in excess of optimum (two per cent or more) the soil was pushed from side to side rather than compacted.

The Harvard Compaction Apparatus proved a very efficient means of obtaining samples for strength tests as well as establishing the moisture vs. dry density relationship. Nevertheless, in order to obtain consistent results from this apparatus it was necessary to have all specimens molded by the same person. The reason for the latter is that the volume of the compaction mold is only 1/454th of a cubic foot and small changes in the particulars of the molding process have a noticeable effect on the dry densities obtained.

As aforementioned, the rate of strain in the slow and medium transient tests was controlled by manipulating the valves of the hydraulic testing machine. In order to insure that the apparatus was traveling at the proper speed an extensiometer mounted on a ring stand was used. This device rested on the turntable of the testing machine, and as the latter moved upward the extensiometer came in contact with the loading head and was depressed. Utilizing a stop watch, the rate at which the depression occurred was determined and this assumed equivalent to the rate of deformation of the specimen.

In the medium transient test the extensiometer was not used. Instead, points two inches apart were marked on the runners of the hydraulic machine, and the time required for the turntable to traverse this distance was noted by a stop watch. Thus the rate determined from

these measurements was considered that at which the specimen was deforming.

The time of loading, instead of the rate of strain, was measured in the fast transient tests. However, this value was converted to rate of strain in order that it might be compared with the other test results. Signals were received from the transformers, by the oscilloscope every 2000th of a second. These signals showed up as dots on the picture of the oscilloscope screen taken during the test. Thus by counting the number of dots on the picture, the time of loading was determined (see Figure 8).

In the fast transient tests it was impossible to control the rate of strain. Consequently, there was a large variance in the rate of strain for the fast transient test specimens. However, since the time of loading was practically the same, for all specimens, it is felt that the fast transient tests results are valid.

It should be noted that the water content of the portion of the specimen in which failure occurs may be different from that of the average moisture content. Consequently, differences in strength between specimens having about the same average moisture content may be due to differences between the moisture content of the portion of the specimen in which failure developed and the average water content of the entire specimen.

However, since the average moisture content is the one used in the field the author considered its determination the most practical. Also, the design of earth structures as well as foundations are based on an average moisture content.

RESULTS

Strength

In both the clay and the silty clay, the strength vs. moisture content curves all exhibited a "peaked" shape, as indicated in Figures 9 and 10. Theoretically the strength should continue to rise as the moisture content decreased. However, there was a tendency for a reduction in this "peaked" condition as the rate of strain increased.

There are two possible reasons for the aforementioned; (a) the condition is the result of the inherent characteristics of the test i.e. at low moisture contents the specimens fail by crumbling rather than shear, due to a lack of lateral confinement, or (b) the condition is a result of the molding process i.e. kneading type compaction imparts this characteristic to the soil.

At the lower moisture contents tested, 9-10 per cent for the silty clay and 14-15 per cent for the clay, the specimens were brittle and would crumble around the edges if not handled carefully. There is great possibility that this tendency existed at higher moisture contents but was just more difficult to detect.

In all cases the maximum strength, as indicated from these tests, occurred at a moisture content less than optimum (Figures 9 and 10). Exactly how much seemed to depend upon the rate of strain and the compactive effort i.e. as the compactive effort decreased there was a tendency for the difference between O.M.C. and the moisture content at

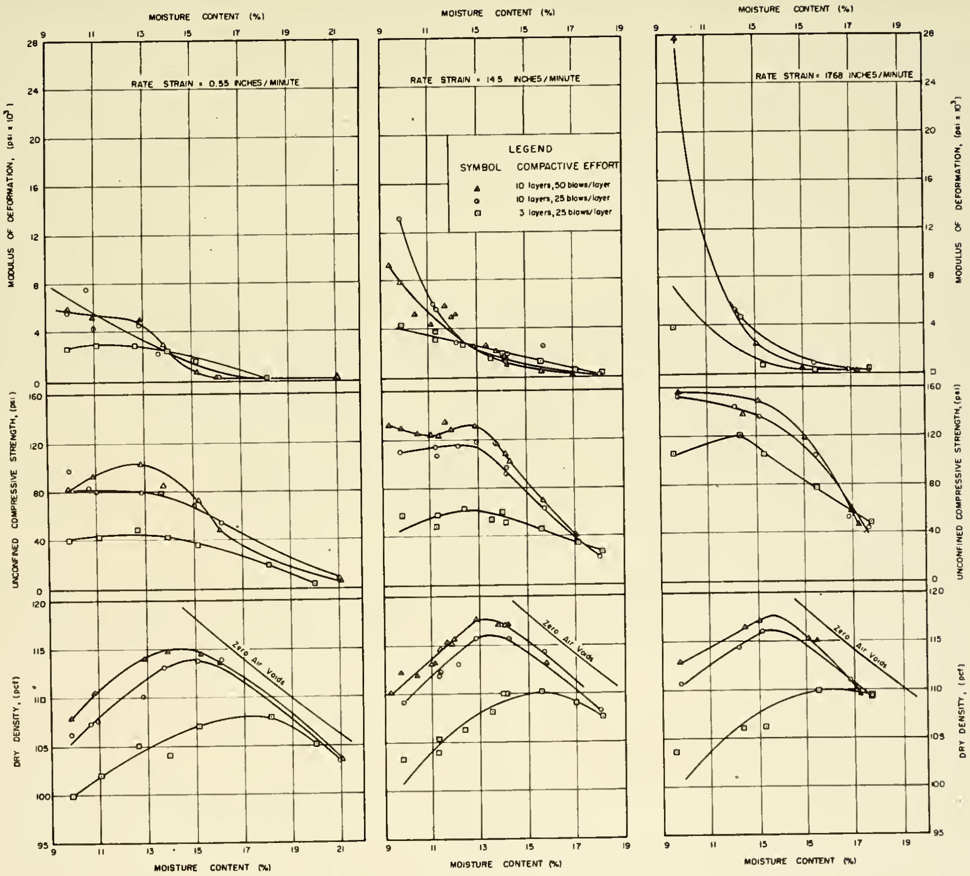


FIG. 9 MOISTURE CONTENT VS. DRY DENSITY, UNCONFINED COMPRESSIVE STRENGTH, AND MODULUS OF DEFORMATION SILTY CLAY

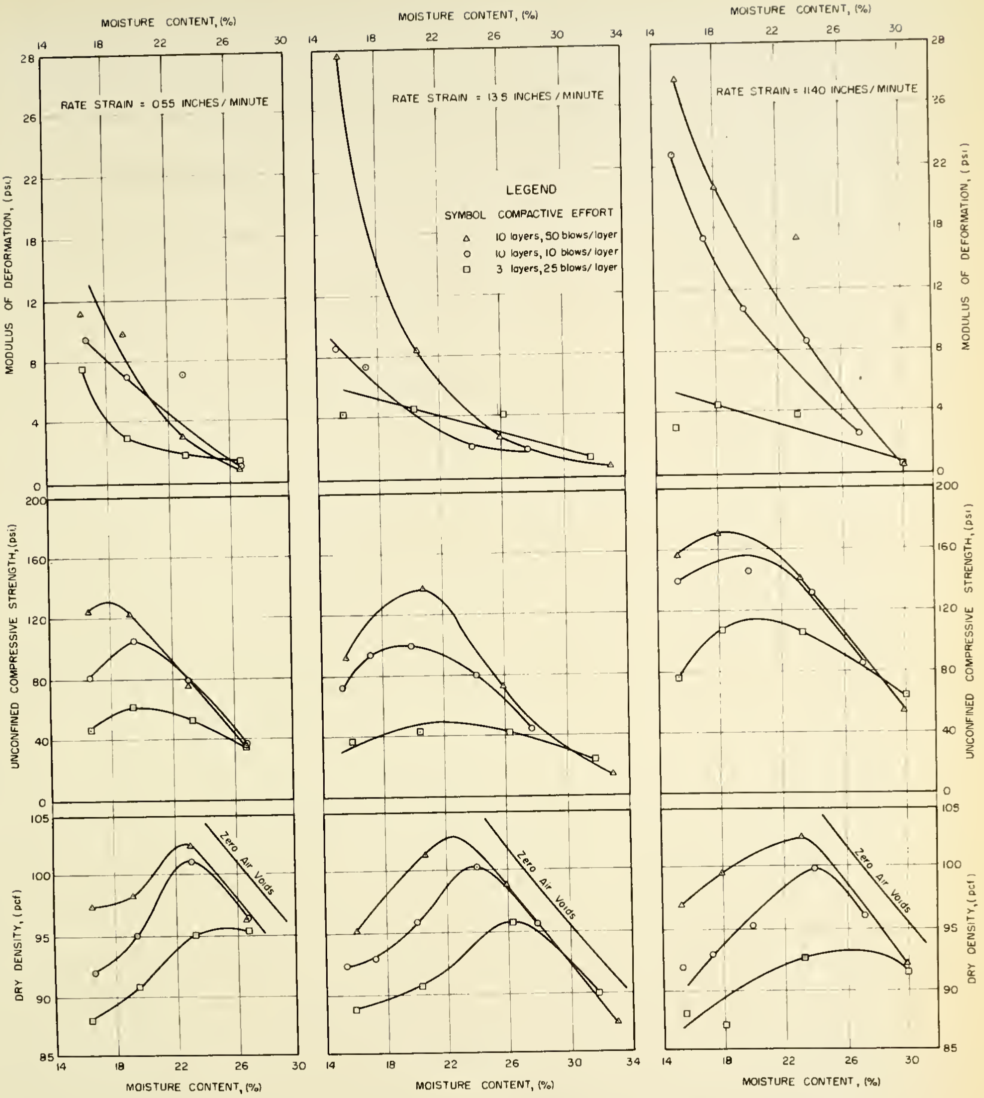


FIG. 10 MOISTURE CONTENT VS. DRY DENSITY, UNCONFINED COMPRESSIVE STRENGTH, AND MODULUS OF DEFORMATION — CLAY

which maximum strength occurred to increase. The maximum difference between the latter two was approximately 4 per cent in the silty clay and 5 per cent in the clay.

Once the moisture content at which the optimum strength was obtained was passed, the change in strength for a given change in moisture content increased significantly (Figures 9 and 10). It appears that the greater the rate of strain the steeper this portion of the curve becomes.

The foregoing is true for both the silty clay and clay. However, in the clay, the rate of increase of strength for a given increase in moisture content on the dry side of optimum strength was equal and in some instances greater than the rate of decrease which occurred on the wet side. Whether or not this is significant depends on the validity of the tests conducted on the dry side i.e. whether or not the specimens were failing principally by crumbling or by shear.

Silty Clay

It was found that, for a given compactive effort and moisture content, increasing the rate of strain produced a measurable increase in the strength of the soils (Figures 11 and 13). In the case of the silty clay, for the lowest compactive effort, an increase of over 100 per cent in the unconfined compressive strength was obtained by increasing the rate of strain from 0.55 in./min. to 1768 in./min., regardless of the moisture content of the sample (Table 3). For the highest compactive effort the increase in strength for the same change in rate of strain ranged between 52 and 87 per cent while for the intermediate compactive effort the range was from 20 to 89 per cent.

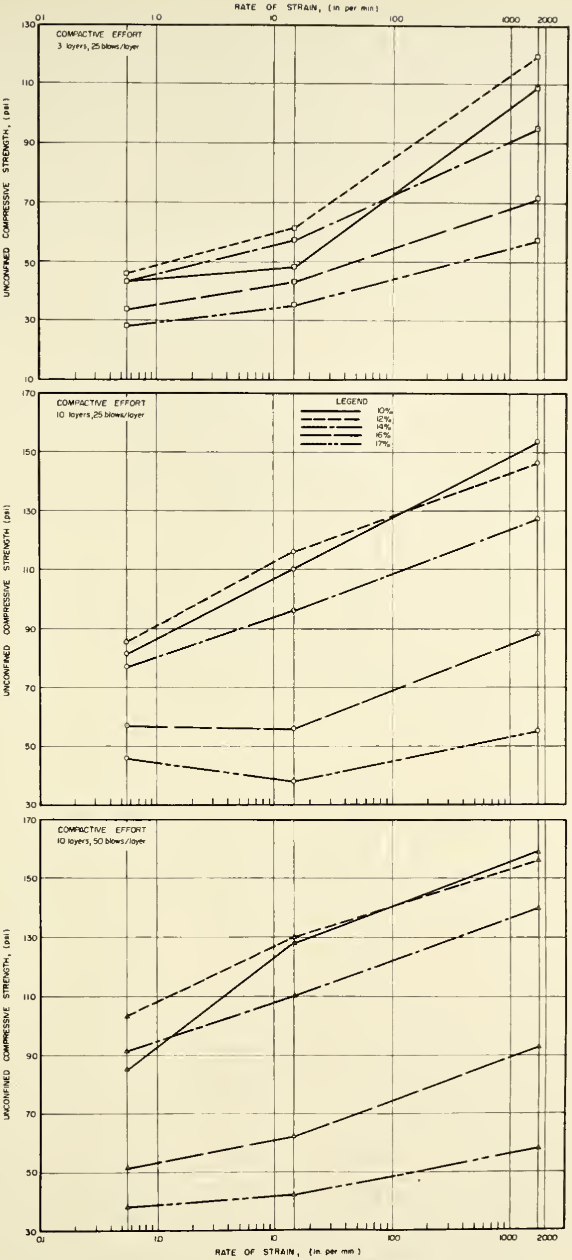


FIG. 11 RATE OF STRAIN VS UNCONFINED COMPRESSIVE STRENGTH — SILTY CLAY

TABLE 3

SUMMARY OF RESULTS OF STRENGTH TESTS - SILTY CLAY

RATE STRAIN	COMPACTIVE EFFORT					
	3 Layers/25 Blows		10 Layers/25 Blows		10 Layers/50 Blows	
	MOISTURE CONTENT = 10%					
	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio
0.55 "/min.	43	1.00	81	1.00	85	1.00
14.5 "/min.	48	1.12	110	1.36	128	1.51
1768 "/min.	109	2.54	153	1.89	159	1.87
	MOISTURE CONTENT = 12%					
	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio
0.55 "/min.	46	1.00	83	1.00	103	1.00
14.5 "/min.	61	1.33	116	1.40	130	1.26
1768 "/min.	119	2.59	146	1.76	156	1.52
	MOISTURE CONTENT = 14%					
	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio
0.55 "/min.	43	1.00	77	1.00	91	1.00
14.5 "/min.	57	1.32	96	1.25	110	1.21
1768 "/min.	95	2.21	127	1.65	140	1.54
	MOISTURE CONTENT = 16%					
	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio
0.55 "/min.	34	1.00	57	1.00	51	1.00
14.5 "/min.	43	1.26	56	0.98	62	1.22
1768 "/min.	71	2.09	88	1.54	93	1.82
	MOISTURE CONTENT = 17%					
	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio
0.55 "/min.	28	1.00	46	1.00	38	1.00
14.5 "/min.	35	1.25	38	0.83	42	1.10
1768 "/min.	57	2.04	55	1.20	58	1.53

As regards the change in strength of the medium transient test with respect to the slow transient test, for a given density and compactive effort, the increase in strength was much less (Table 3). Considering the lowest compactive effort the increase in strength ranged from 12 to 32 per cent, for the intermediate compactive effort the change ranged from -17 to 40 per cent, and for the highest compactive effort the change in strength was from +10 to +51 per cent.

There appears, on the basis of the data collected, to be a general trend toward a decrease in the effect of rate of strain as the moisture content increases, for a given compactive effort (Figure 11 and Table 3). This, in all probability, is due to the greater effect of pore water pressures which would tend to decrease the strength of the sample. This could also be the reason why the strength ratio values tend to decrease as the compactive effort increases.

From the foregoing discussion, it can be seen that the effect of rate of strain is more significant in increasing the strength of material at low densities. In comparing the fast and slow transient tests, over 100 per cent increase in strength was obtained for the lowest compactive effort. Considering the same range for the other two compactive efforts the maximum increase was 89 per cent, and for the majority of cases it was much less.

Furthermore, as the compactive effort increased the effect of an increase in moisture, on the strength, increased for a given compactive effort (Figure 11). As aforementioned, this is due to an increase in pore pressure brought about by a reduction in void ratio due to increased compaction.

From Figure 12, which is a plot of unconfined compressive strength vs. dry density, at constant moisture content, it can be qualitatively stated that the effect of increased density decreases as rate of strain increases. For example, it can be noted that the slopes of the curves for rate of strain equal to 1768 inches per minute are less than those for either 0.55 or 14.5 inches per minute. Also, moisture content did not effect the slopes appreciably.

Clay

For the clay, it was also found that for a given compactive effort and moisture content, increasing the rate of strain produced a measurable increase in the strength of the soil (Figure 13). As with the silty clay, the strength ratio, for all moisture contents, was greatest for the lowest compactive effort (Table 4). Also, from Figure 13 it can be seen that moisture content has less effect on the strength, at a given rate of strain, the lower the compactive effort i.e. as the compactive effort is increased the effect of moisture content increases.

Table 4 shows that as the moisture content increases the effect of rate of strain on increasing the strength increases for a given compactive effort. However, it should be noted that this relationship is generally valid for only the dry side of optimum since the data on the wet side is incomplete. Nevertheless, by extrapolation of the data obtained it can be safely assumed that the relationship is valid on the wet side of optimum moisture content--at least within a few per cent of this value.

Table 4 also shows that some soils decrease in strength with an increase in rate of strain. This tendency existed at all except one

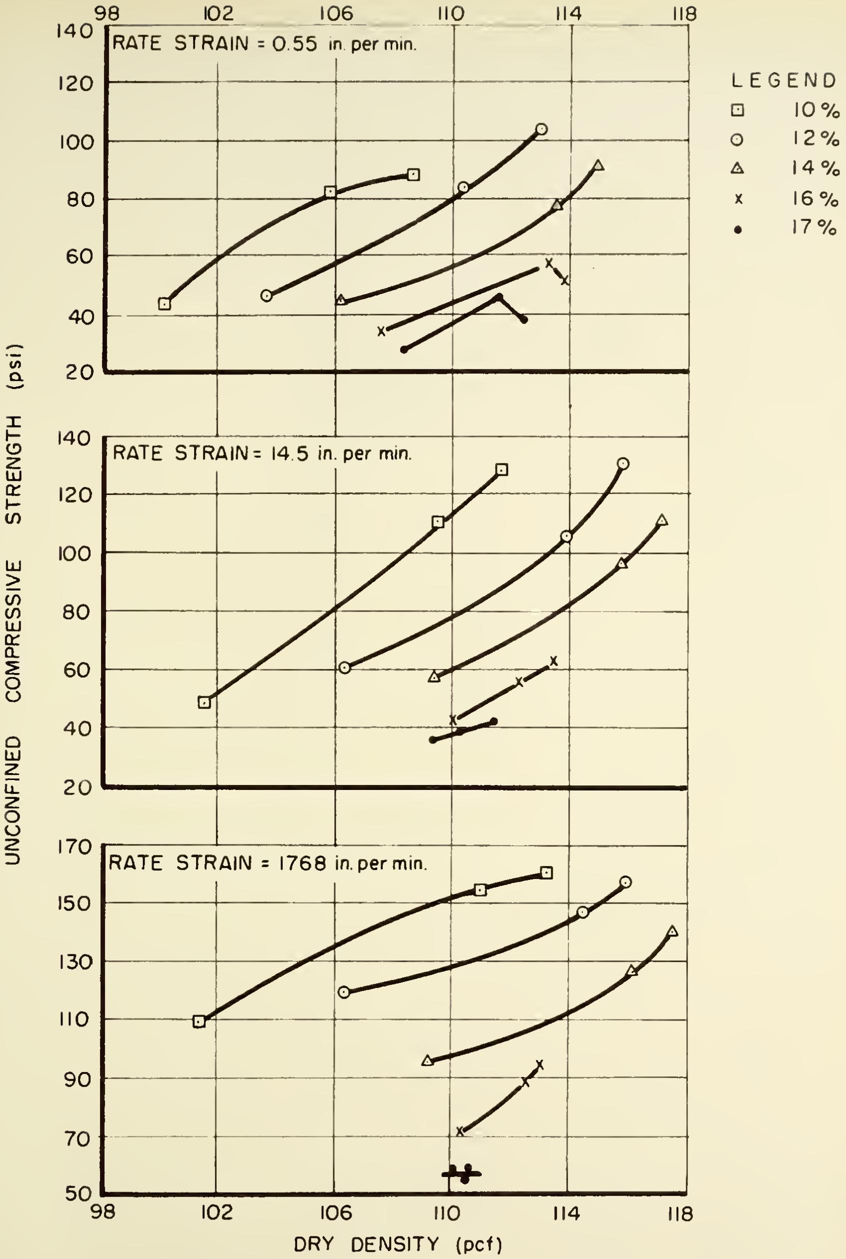


FIG. 12 DRY DENSITY VS. UNCONFINED COMPRESSIVE STRENGTH-SILTY CLAY

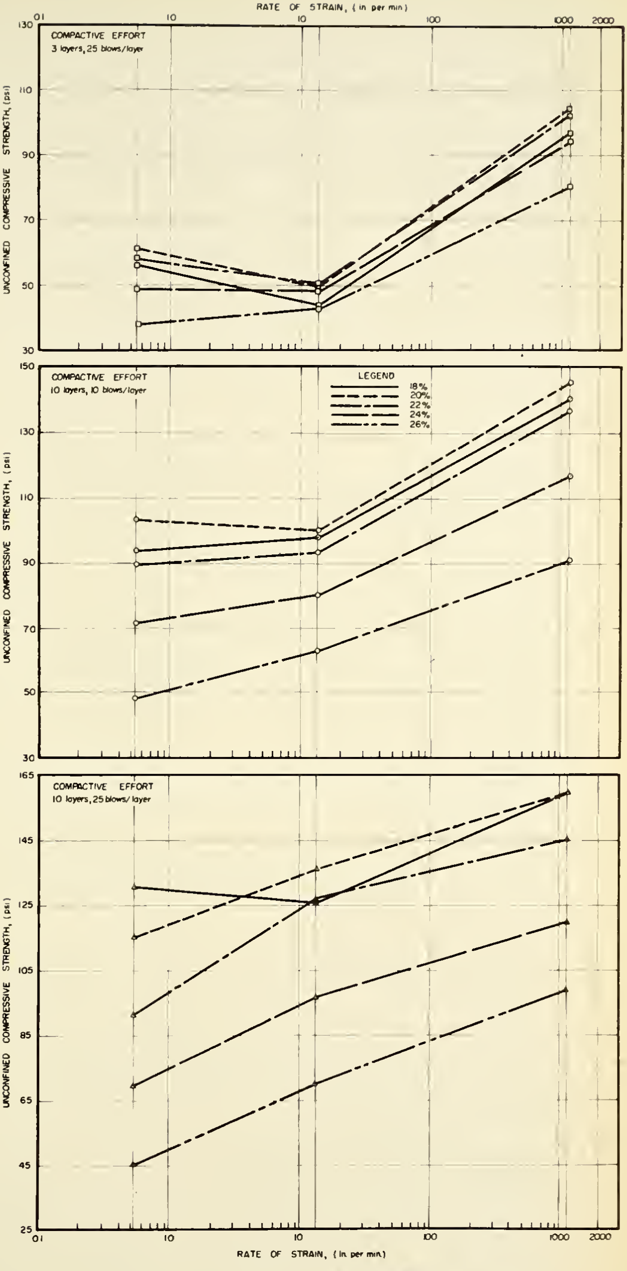


FIG. 13 RATE OF STRAIN VS. UNCONFINED COMPRESSIVE STRENGTH — CLAY

TABLE 4

SUMMARY OF RESULTS OF STRENGTH TESTS - CLAY

RATE STRAIN	COMPACTIVE EFFORT					
	3 Layers/25 Blows		10 Layers/25 Blows		10 Layers/50 Blows	
	MOISTURE CONTENT = 18%					
	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio
0.55 "/min.	56	1.00	94	1.00	131	1.00
13.5 "/min.	44	0.79	98	1.04	126	0.96
1140 "/min.	97	1.73	142	1.51	160	1.22
	MOISTURE CONTENT = 20%					
	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio
0.55 "/min.	61	1.00	103	1.00	115	1.00
13.5 "/min.	49	0.80	100	0.97	136	1.18
1140 "/min.	104	1.71	146	1.42	160	1.39
	MOISTURE CONTENT = 22%					
	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio
0.55 "/min.	58	1.00	89	1.00	93	1.00
13.5 "/min.	50	0.86	93	1.04	128	1.36
1140 "/min.	102	1.76	137	1.54	146	1.57
	MOISTURE CONTENT = 24%					
	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio
0.55 "/min.	49	1.00	71	1.00	69	1.00
13.5 "/min.	48	0.98	80	1.13	98	1.42
1140 "/min.	92	1.88	117	1.65	120	1.74
	MOISTURE CONTENT = 26%					
	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio	Strength (psi.)	S_u Ratio
0.55 "/min.	38	1.00	48	1.00	45	1.00
13.5 "/min.	43	1.13	63	1.31	70	1.56
1140 "/min.	80	2.11	91	1.90	95	2.11

moisture content, the highest, at the lowest compactive effort. In the two higher compactive efforts the condition existed at only one moisture content, in each. Since the latter values are so close to unity, it could be assumed that there was not a decrease and the stress ratio is unity.

However, due to the fact that the stress ratios for the medium transient tests are consistently less that 1.0 this might be a significant trend, for the lowest compactive effort. It can also be noted that a value of the stress ratio greater than 1.0 occurs at nearly the optimum moisture content. This indicates that on the wet side, stress ratios greater than 1.0 would be obtained. Also, it must be recognized that since the stress ratio for the medium transient tests are generally very close to unity, at low moisture contents and low compactive efforts, and since an average curve is drawn through the data obtained this could account for the stress ratio being less than 1.0.

A study of Figure 13 reveals that the higher the compactive effort the greater the effect of a change in moisture content on the strength of the clay soil. For example, as the compactive effort is increased the curves in Figure 13 become farther apart.

It should be noted that the 18 per cent moisture content curve in Figure 13 is out of line with the remainder of the data. This is probably because the specimens tested at this moisture content failed, partially at least, by crumbling.

Figure 14 illustrates, qualitatively, that the effect of an increase in density, decreases as the rate of strain increases. This can be noted from the fact that the slopes of the curves for rate of strain

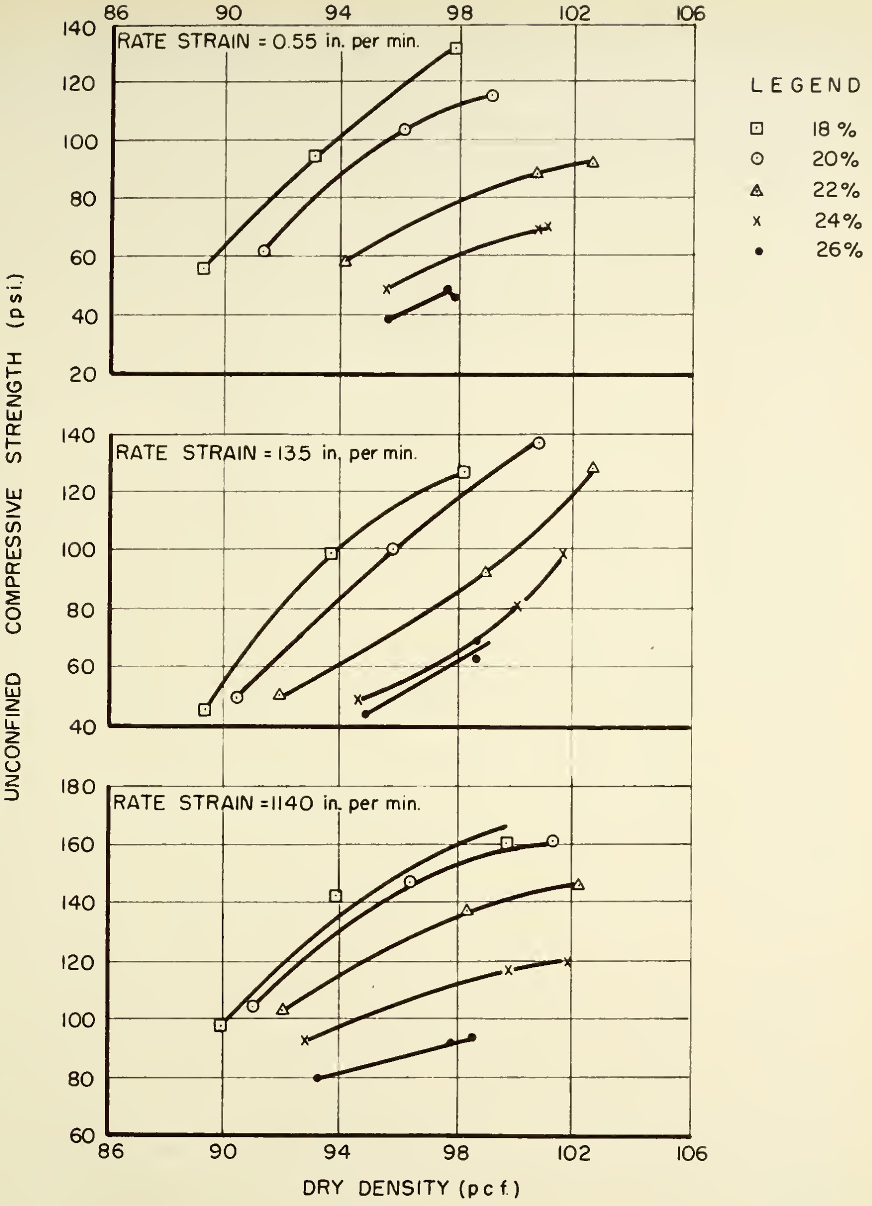


FIG. 14 DRY DENSITY VS. UNCONFINED COMPRESSIVE STRENGTH - CLAY

equal to 1140 inches per minute are less than those for either 0.55 or 13.5 inches per minute.

It is worthy of note that the strength ratio for the clay increases with increasing moisture content while that for the silty clay decreases with increasing moisture content. Also, for a given rate of strain the effect of moisture content on the strength is greater for the silty clay than the clay. However, the latter trend decreases as the compactive effort decreases.

Modulus of Deformation

The data on the modulus of deformation, for both soils tested, was the most difficult to interpret. This was due to the large scattering of the values (see Figures 9 and 10). The latter situation being caused by the difficulty in picking values from the oscilloscope trace, as shown on the photograph of same (Figure 8). It was quite easy to determine the maximum load, on the specimen, and the strain at which it occurred. However, at low loads the ability to accurately determine the load on the specimen, and the strain at which it occurred was greatly reduced. As a result, in practically all cases, the initial portions of all stress strain curves appeared to be out of line.

Consequently, the modulus of deformation vs. moisture content curves, for a given compactive effort and rate of strain can only be used to obtain general trends as indicated by the results shown (Figures 9 and 10). Since the modulus of deformation vs. rate of strain curves (Figures 15 and 16), for a given moisture content and compactive effort, were obtained from the curves previously indicated they, also, are only of qualitative value.

Silty Clay

Regardless of the scattering of the points in Figure 9 it can be readily seen that moisture content has a tremendous effect on the modulus of deformation of the silty clay. It is also apparent that as the rate of strain increases the effect of moisture on the modulus of deformation also increases.

This rate of change of modulus of deformation with respect to a change in moisture content decreases as the moisture content increases until a point is reached where the compactive effort has a negligible effect on the modulus of deformation. This condition occurs on the dry side of optimum for the slow and medium transient tests, for all compactive efforts, and approximately at optimum for the fast transient tests.

In fact, if a soil is to be compacted at a moisture content which approximates the O.M.C. for standard compaction, considering the fast transient test conditions, it appears that use of a compactive effort greater than Standard Proctor is not warranted. Considering the fact that a subgrade usually gains approximately 2 per cent moisture before coming to equilibrium with its natural surroundings, this would make compaction at an effort greater than standard AASHO impractical, unless methods of preventing an increase in moisture content were provided. The latter statement is assuming that the modulus of deformation is to be used as the criteria of strength.

Considering the moisture content range tested it appears a significant increase in modulus of deformation due to an increase in rate of strain was obtained only for the specimens compacted at about 10 per

cent moisture (Figure 15). This is true regardless of the compactive effort. Also, on the basis of Figure 15 it can be easily seen that moisture content has a much more significant effect on the modulus of deformation than does rate of strain.

As regards moisture contents of 12 per cent or above it appears that an increase in the rate of strain will not produce an increase in the modulus of deformation. From Figure 15 and Table 5 it can be seen that for certain moisture contents there appears to be a decrease in the modulus of deformation with an increase in the rate of strain. This can probably be attributed to the scattering of data. Also, the scattering of data might be the reason there was a significant increase in modulus of deformation with rate of strain for the 10 per cent moisture content specimens.

Table 5 indicates an important trend. For a given compactive effort and rate of strain the M_D ratio decreases with increasing moisture content. There was, however, too much variability of data to make quantitative comparisons between various compactive efforts.

Clay

In the clay soil there was also a substantial decrease in the modulus of deformation for a given increase in moisture content at a given rate of strain (Figure 10). This condition was more pronounced the higher the compactive effort and the greater the rate of strain. On the basis of the data given, at moisture contents approximating the optimum moisture content for Standard AASHO the effect on the modulus of deformation of an increase in compactive effort is negligible. However, as the rate of strain increases the effect of the amount of com-

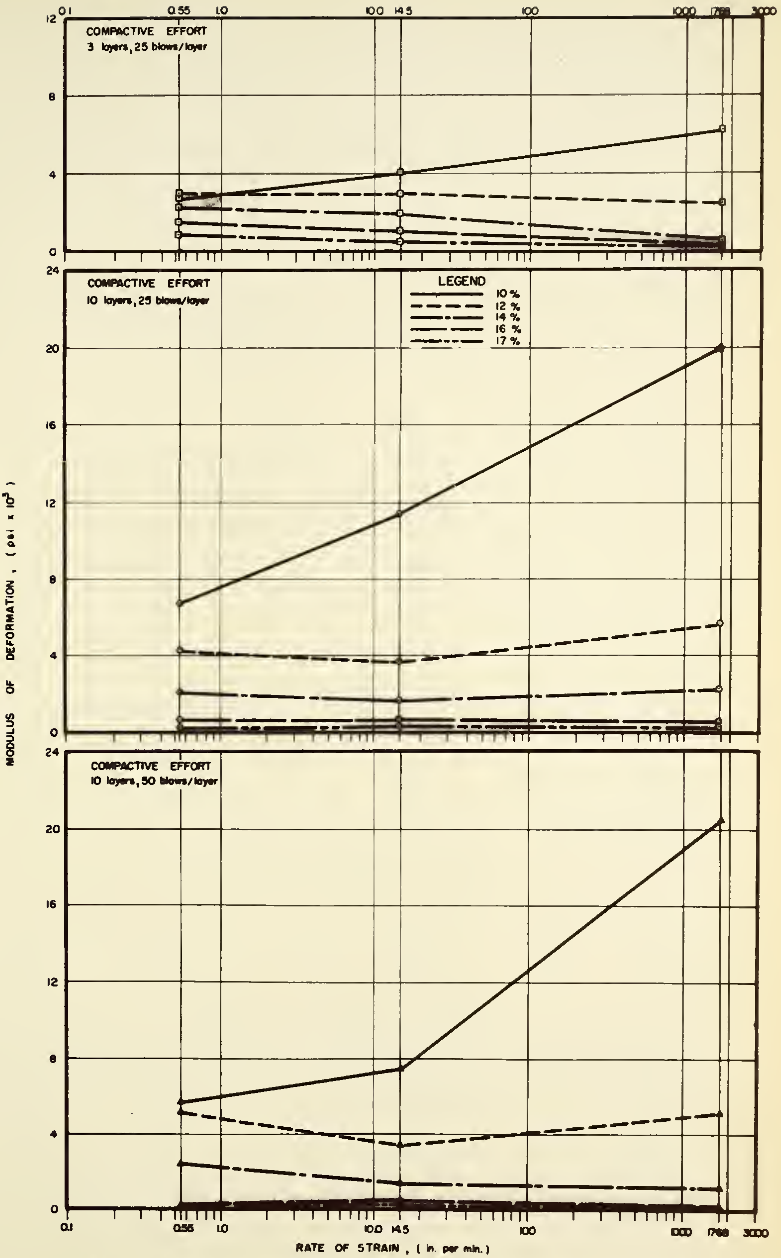


FIG. 15 RATE OF STRAIN VS. MODULUS OF DEFORMATION — SILTY CLAY

TABLE 5

SUMMARY OF MODULUS OF DEFORMATION DATA - SILTY CLAY

RATE STRAIN	COMPACTIVE EFFORT					
	3 Layers/25 Blows		10 Layers/25 Blows		10 Layers/50 Blows	
	MOISTURE CONTENT = 10%					
	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio
0.55 "/min.	2700	1.00	6700	1.00	5700	1.00
14.5 "/min.	3900	1.46	11,300	1.69	7400	1.30
1768 "/min.	6200	2.30	20,000	3.22	20,600	3.61
	MOISTURE CONTENT = 12%					
	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio
0.55 "/min.	2900	1.00	4200	1.00	5200	1.00
14.5 "/min.	2950	1.02	3600	0.86	3400	0.65
1768 "/min.	2400	0.83	5600	1.33	5200	1.00
	MOISTURE CONTENT = 14%					
	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio
0.55 "/min.	2200	1.00	2050	1.00	2500	1.00
14.5 "/min.	1950	0.88	1600	0.78	1400	0.56
1768 "/min.	550	0.25	2250	1.10	1250	0.50
	MOISTURE CONTENT = 16%					
	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio
0.55 "/min.	1040	1.00	600	1.00	150	1.00
14.5 "/min.	1000	0.96	600	1.00	500	3.33
1768 "/min.	250	0.24	550	0.92	150	1.00
	MOISTURE CONTENT = 17%					
	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio
0.55 "/min.	800	1.00	200	1.00	75	1.00
14.5 "/min.	450	0.56	350	1.75	200	2.67
1768 "/min.	200	0.25	200	1.00	50	0.67

pactive effort becomes apparent.

From Figure 16 and Table 6 the effect of rate of strain and moisture content on the modulus of deformation can be observed. For all compactive efforts and rates of strain there was a tendency toward an increase in modulus of deformation with an increase in rate of strain. In a few instances, particularly the medium transient tests at the intermediate compactive effort, there was a slight decrease in the modulus of deformation with an increase in rate of strain. This can be attributed to the scattering of the data, and the attempt to draw a representative curve through it. The aforementioned reason was also the cause of the M_D ratios being less than one, in all probability (Table 6).

Consequently, it can be stated that for a given rate of strain the M_D ratio increases with increasing moisture content. Also, the M_D ratio, for a given moisture content and the fast transient test conditions, increased as the compactive effort increased. This tendency was not apparent in the M_D ratio for the medium transient tests.

In the case of the intermediate effort, considering the medium transient tests, the value of the M_D ratio obtained was much less than either of the other two compactive efforts. Also, the caliber of the data did not produce results from which a general trend could be predicted for the medium transient tests i.e. no definable tendency of the variance of the M_D ratio of the medium transient tests at a given moisture content but varying compactive effort.

As can be seen from Figure 16 moisture content has an increasingly important effect on the modulus of deformation the higher the compactive effort. Also, it is shown that significant increases in the modulus of deformation may be obtained by increasing the rate of strain. It should

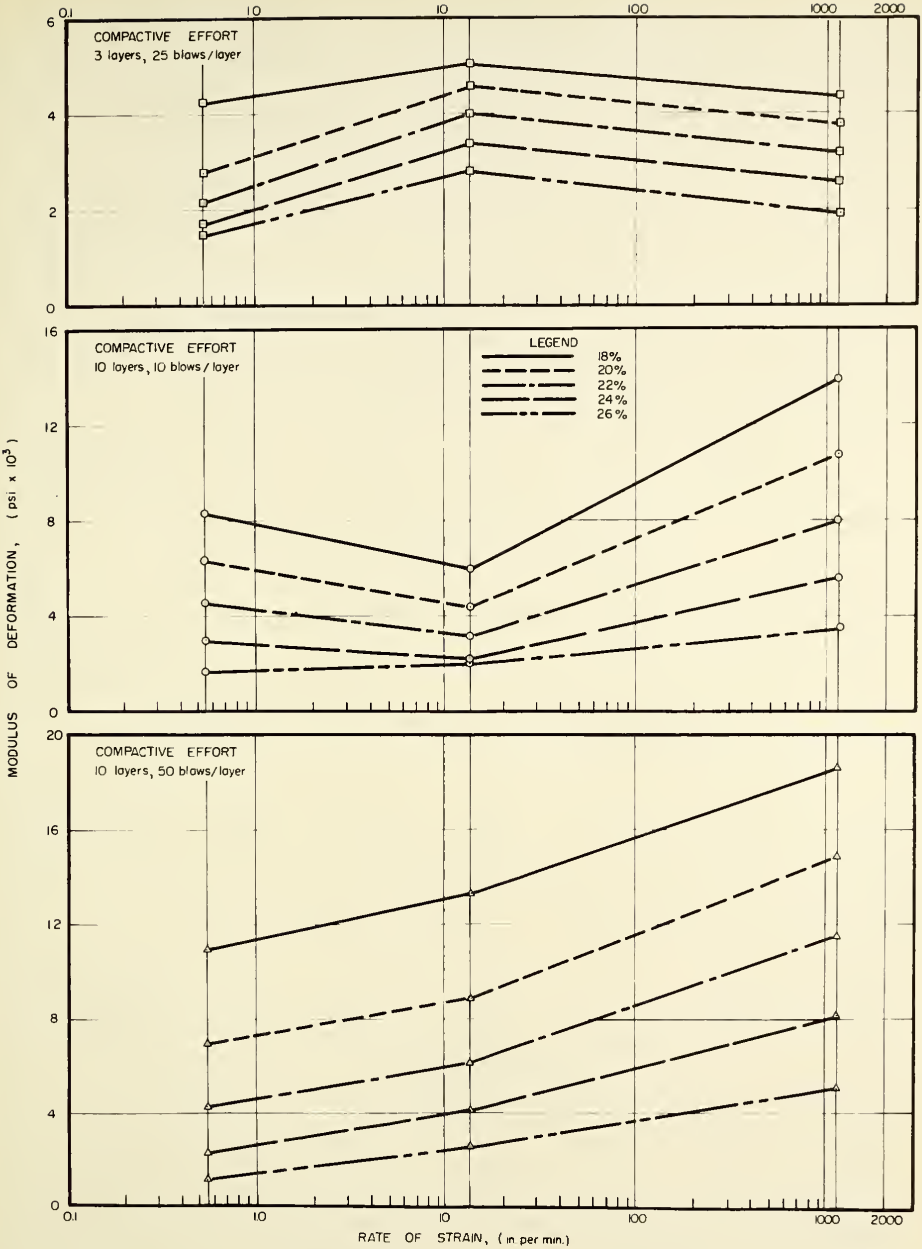


FIG. 16 RATE OF STRAIN VS. MODULUS OF DEFORMATION —— CLAY

TABLE 6

SUMMARY OF MODULUS OF DEFORMATION DATA - CLAY

RATE STRAIN	COMPACTIVE EFFORT					
	3 Layers/25 Blows		10 Layers/25 Blows		10 Layers/ 50 Blows	
	MOISTURE CONTENT = 18%					
	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio
0.55 "/min.	4300	1.00	8300	1.00	10,900	1.00
13.5 "/min.	5100	1.19	6000	0.72	13,400	1.23
1140 "/min.	4400	1.02	13,900	1.68	18,700	1.71
	MOISTURE CONTENT = 20%					
	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio
0.55 "/min.	2800	1.00	6300	1.00	7000	1.00
13.5 "/min.	4600	1.64	4400	0.70	8900	1.27
1140 "/min.	3800	1.36	10,700	1.70	15,000	2.14
	MOISTURE CONTENT = 22%					
	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio
0.55 "/min.	2200	1.00	4500	1.00	4200	1.00
13.5 "/min.	4000	1.80	3100	0.69	6200	1.48
1140 "/min.	3200	1.45	8000	1.78	11,600	2.76
	MOISTURE CONTENT = 24%					
	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio
0.55 "/min.	1700	1.00	3000	1.00	2300	1.00
13.5 "/min.	3400	2.00	2200	0.73	4100	1.78
1140 "/min.	2600	1.53	5600	1.87	8300	3.61
	MOISTURE CONTENT = 26%					
	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio	M_D(psi.)	M_D Ratio
0.55 "/min.	1500	1.00	1600	1.00	1200	1.00
13.5 "/min.	2800	1.87	2000	1.25	2600	2.17
1140 "/min.	1900	1.27	3600	2.25	5300	4.41

be noted that the higher the compactive effort the greater the effect of rate of strain on M_D.

For the lowest compactive effort, a decrease in M_D for an increase in rate of strain was noted (Figure 16). This is probably due to the scattering of data and the use of average curves.

Comparison of Clay and Silty Clay

In both the clay and silty clay, a significant increase in unconfined compressive strength occurred with an increase in rate of strain for all compactive efforts and all moisture contents tested. However, with the silty clay, as the moisture content increased, the strength ratio decreased while for the clay the converse was true.

In both the clay and the silty clay the strength ratios for the fast transient test were greater for the lowest compactive effort. No definite relationship could be established for the medium transient tests between the strength ratios and the compactive effort.

In both the clay and the silty clay the moisture content proved to be more significant than the increase in rate of strain, as regards affecting the unconfined compressive strength. This tendency became less as the compactive effort decreased.

Moisture content has a great effect on the modulus of deformation of the silty clay as well as the clay. This condition was more pronounced the higher the compactive effort and the higher the rate of strain. Furthermore, for the clay soil, increasing the rate of strain produced a measurable increase in the M_D ratio, but this was not the case for the silty clay.

It is worthy to note that the data presented in this report does not give a straight line relationship, on a semi-logarithmic plot, for the variance of strength or modulus of deformation with respect to rate of strain (Figures 11, 13, 15, and 16). However, the validity of the shape of the strength vs. rate of strain curves contained in this report are substantiated by those in Reference (23).

As regards the utility of the strength ratio and modulus of deformation ratio, it should be recognized that these are percentages. Consequently, it takes much less of an increase in strength to produce a strength ratio of 2 for a soil of low strength, as determined from a slow transient test, as one of high strength determined from the same test. For example, the numerical increase in strength of a dry sample at the highest compactive effort can be much greater than that of a sample at the same moisture content but the lowest compactive effort, and yet its strength ratio can be lower.

In contrast to Whitman (23), this report shows a general trend toward a decrease in the strain at failure with an increase in the rate of strain. This is attributed to the fact that the faster the rate of strain the faster the load is applied which results in the specimen not having the opportunity of deforming greatly before it developes its maximum shear strength.

Finally, it is of interest to investigate the possibility of thixotropic action affecting the results, since the specimens were tested approximately 24 hours after molding. Both Moretto (24) and Skempton and Northey (25) observed that thixotropic strength regain decreased with decreasing water content below the liquid limit. Also, Seed and Chan (26) showed that thixotropic effects are relatively small,

for samples compacted on the dry side of optimum for the compactive effort being used.

Consequently, since most of the specimens were compacted on the dry side of optimum, since those compacted on the wet side were well below the liquid limit, and since the time of storage was a maximum of twenty-four hours, thixotropic effects were not significant in this study.

CONCLUSIONS

On the basis of these tests it can be concluded that:

1. As the compactive effort is increased the greater the effect of changes in moisture content on the unconfined compressive strength, regardless of the rate of strain.
2. The lower the compactive effort the greater the effect of rate of strain on the strength.
    a. Considering the silty clay, as the moisture content increases the strength ratio tends to decrease.
    b. Considering the clay, as the moisture content increases the strength ratio increases.
3. As the rate of strain increases the effect of an increase in dry density, at a given moisture content, on the unconfined compressive strength decreases.
4. Increasing the rate of strain produced a measurable increase in the M_D ratio of the clay soil.
    a. For the clay soil, as the moisture content increased the M_D ratio increased for a given compactive effort but the highest compactive effort yielded the greatest increase.
    b. In the silty clay soil there was a tendency toward a decrease in M_D ratio for a given rate of strain and compactive effort.
    c. Due to the scattering of data it was impossible to

compare the M_D ratios of the various compactive efforts and establish a trend.

5. On the basis of these tests it must be concluded that to obtain significant increases in strength or modulus of deformation it takes a rate of strain equivalent to a fast transient test.
6. As the rate of strain decreases there is a tendency for the strain at failure to increase.

DISCUSSION OF POSSIBLE APPLICATIONS

Stability of Slopes Subjected to Earthquakes

Earthquakes have caused shear slides in soft and brittle rocks, loess and uncemented loose deposits. There are two possibilities as to why such a failure would occur: (1) the vibrations caused a rapid reduction in void ratio which, since the soils were sufficiently saturated in certain portions, produced liquefaction or (2) during collapse of the soils structure, the pressures were temporarily carried by the large volume of air in the soils. In the latter instance a considerable time will be required for the excess air to escape, and during this period the shear strength of the material would be reduced to a small value (1).

Conversely, however, there is no evidence that earthquakes have caused shear slides in natural slopes or embankments consisting of clay or underlain with clay (1). Under normal static conditions such failures are common, and one would expect that slopes or embankments which are close to shear failure have been exposed to earthquakes and nevertheless have not failed. This evidence supports the conclusions of this study which found that the transient shear strength of a clay is sufficiently greater than static value to maintain at least the same factor of safety during an earthquake as exists before.

Casagrande and Shannon (1), on the basis of their test results as well as all other information available, concluded:

1. Very steep slopes consisting of weak, brittle rocks or loess, are vulnerable to transient loading caused by shocks from

earthquakes or explosions. Slides resulting in such materials may be seriously aggravated by liquefaction phenomena.

2. Deposits of loose, saturated sand or mountain detritus may be liquefied by shocks due to earthquakes or explosions.

3. Non-cohesive material in a dry and loose state, which is sloping at the angle of repose, may be affected by shocks to the extent that the slope is slightly flattened.

4. Well compacted, non-cohesive materials, either in a dry or saturated state, which are placed in embankments with customary slopes, may be considered safe against slides due to transient loading by earthquakes or explosions.

5. Many soft, non-plastic or slightly plastic silts (both organic and inorganic types) which are often called mud, have a slightly sensitive structure and may liquify under the effects of seismic disturbances or explosions.

6. Relatively flat slopes in clays or in Cucaracha rock, and clay deposits underlying embankments, which are safe under static conditions, may also be considered safe against shear slides under transient loading due to earthquakes or explosions.

7. Clay strata underlying an embankment consisting of loose sand with slopes close to the angle of repose, may receive a substantial increase in shear stresses due to partial loss of stability in the interior of the embankment. This increase may be greater than the transient strength of the clay and, therefore, may cause a shear slide through the clay foundation.

On the basis of library research and the data presented in this report the above conclusions appear to be warranted.

Applications to Highway and Airport Design

As has been previously mentioned, the character of the speed of travel on various sections of highway and airport pavements varies greatly. Therefore, in order to obtain the most economical as well as the most structurally adequate pavements, it is necessary to evaluate the effect of speed of travel on the stress-strain characteristics of these pavements. The data show the effect of transient loading on the unconfined compressive strength to be dependent upon the type of load,

type of soil, and the moisture content and dry density of the soil. Thus, if the effect of rate of strain is to be adequately interpreted, the aforementioned variables must be evaluated.

Structural failure in flexible pavements may be due either to excessive shear stresses, excessive deformation brought about by consolidation, or fatigue failures. For flexible pavements, shear failures are quite common. However, excessive deformation caused by repeated loads also result in considerable distress.

Since shear failures are caused by exceeding the shearing resistance of the soil, and since it has been determined, in this study, that the shear strength may be increased considerably when soils are subjected to transient loads, the chance of shear failures in pavements designed on the basis of slow tests, but which are subjected to transient loads are reduced. Following the same reasoning, considering shear failures which occur at the edges of flexible pavements due to the high concentration of stress at these points, better performance should be obtained from sections of the pavement where travel is free flowing than at intersections and other points along the route where vehicles are required to proceed slowly or to stop.

Where shear is the major consideration high densities are worthwhile if moisture content can be controlled. Therefore, in order to take full advantage of high dry densities brought about by increased compaction, the pavement must be provided with a good drainage system.

Due to the fact that limiting deflection rather than ultimate strength is generally the criteria for design of highway and airport pavements the effect of rate of strain on the modulus of deformation is the critical factor. The data indicate that for silty clay soils,

slow tests can be used to determine the modulus of deformation, but if this method is employed in the determination of the modulus of deformation of a clay misleading results may be obtained. The error, however, will be on the conservative side.

On the basis of the data obtained, it can be said that as the plasticity increases the effect of rate of strain on the modulus of deformation increases. From the information presented, it is apparent that the effect of rate of strain on the modulus of deformation of silty clay is small, and when this material has a high density and high moisture content, it is negligible.

For clay soils, as indicated above, an increase in the rate of strain results in an increase in the modulus of deformation. This indicates that pavements designed on clay soils, which will be subjected to fast moving loads, might best be analyzed on the basis of transient tests. However, as with the silty clay, when high densities are to be used an efficient means of controlling the moisture content must be employed.

This study also shows that, for certain soils, as the rate of strain is increased the character of the soil (physical properties) become less important and strength is principally a function of the rate of strain and the moisture content. This indicates that if moisture content is controlled soils of lower quality can be utilized for subgrades under pavements where fast moving loads are anticipated. This is based upon the assumption that there are no other properties of the soil in question which would render it undesirable i.e. very elastic material.

Also, for properly designed pavements the deflections resulting from fast moving loads are less than those for the same material subjected to slow moving loads. This would require more repetitions of load to cause fatigue failure in pavements subjected to fast moving loads. Along this same line of reasoning, it can be said that pavements subjected to fast moving loads should pump less than those subjected to slow moving loads because the deflection of the pavement in the latter instance will be greater.

In summary, it should be emphasized that the use of transient load tests for the design of structures subjected to fast moving loads may result in a great saving in time and money. However, where an extensive testing program is not feasible some of the items enumerated above may be qualitatively taken into consideration.

RECOMMENDATIONS FOR FURTHER RESEARCH

1. It is recommended that this research be extended to include silts.
2. More work is needed on partially saturated clays and silty clays.
3. It would also be extremely worthwhile to conduct triaxial tests, extending from slow to fast transient speeds, on the aforementioned soils, over a range of moisture contents and densities.
4. Finally, it is recommended that this research be extended to include undisturbed samples of the aforementioned soils. It should be of great significance to note the variance, if any, of the results of the tests on undisturbed and disturbed samples for the same soils.

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